

#68-F-228M

INKERMAN WEST

BRIDGE

MOUNTAIN

TWP.

BA 2865
Site 31-78

H. Q. GOLDER & ASSOCIATES LTD.

SOIL AND FOUNDATION ENGINEERS

HEAD OFFICE - TORONTO, ONTARIO

H. Q. GOLDER

V. MILLIGAN

L. G. SODERMAN

J. L. SEYCHUK

F. J. HEFFERNAN (OTTAWA)

196 BRONSON AVENUE

OTTAWA 4, ONTARIO

235-9698

July 17, 1968.

Graham, Berman and Associates Ltd.,
Consulting Engineers,
2277 Riverside Drive,
Ottawa 8, Ontario.

Attention: Mr. A. R. Sethna, P. Eng.

RE: Soil Investigation,
Proposed Inkerman West Bridge,
Township of Mountain.

Dear Sirs:

This letter reports the results of an investigation carried out at the bridge crossing of the north branch of the South Nation River by a township road, 1 mile west of Inkerman. The purpose of this investigation was to determine the subsoil and groundwater conditions at the site and, based on this information, to make recommendations for the foundation design of a proposed bridge replacement structure.

PROCEDURE

The field work for this investigation was carried out on June 26 and 27, 1968. A borehole was put down through the east approach embankment. The bouldery nature of this approach embankment required the use of diamond drilling methods to advance the casing through this embankment fill. A dynamic penetration test was put down through the west approach embankment to some 15 feet below the stream bed. The boring and the dynamic penetration test were put down with a machine drill rig supplied and operated by the F. E. Johnston Drilling Co. Ltd, Ottawa. The field work was supervised by a member of our engineering staff.

The location of the boring and the penetration test, together with a stratigraphic section along the centerline of the proposed bridge, are shown on Figure 1. A detailed log of the borehole is given on the Record of Borehole sheet following the text of this report.

The soil samples were brought to our laboratory for detailed examination. The results of a laboratory test on a sample of the till overburden are shown on Figure 2.

The elevations given in this report are referred to a bench mark located on the top of the large boulder, north east of the bridge. The elevation of this bench mark was given to us as 249.25, as referred to Geodetic datum.

SITE AND GEOLOGY

The site is located one mile west of Inkerman at the crossing of the north branch of the South Nation River by the township road which runs westward from Inkerman. The topography of the area is relatively flat. The stream has cut a valley some 10 feet deep at the bridge crossing.

From available geologic information the stream in this area skirts the northern edge of a low drumlin. The bedrock in the area is limestone of the Oxford formation.

SUBSURFACE CONDITIONS

The detailed soil stratigraphy encountered in the borehole is given on the Record of Borehole sheet. Following is a summarized account of the soil conditions.

Embankment Fill

The borehole put down through the east approach embankment encountered some 12 feet of fill which is predominately a sand and gravel material with numerous cobbles and boulders. The fill also contained some silt size material and occasional pockets of clay. The coarse nature of the fill provided a variable resistance to driving the sampler, though it is considered that the fill is in a compact state of packing.

Alluvium

At the borehole location, the embankment fill is underlain

by a 1 foot layer of silty clay which contains some organic material. This layer is probably a geologically recent deposit of the stream, that is, alluvium.

Glacial Till

The alluvium layer is underlain by a stratum of sandy silt till, the principal stratum at the site. The borehole was terminated in the till stratum after penetrating some 18 feet into this deposit. A grain size distribution test was carried out on a representative sample of the till and the results are shown on Figure 2.

Standard penetration tests carried out in the till gave "N" values ranging from 14 blows/ft near the surface to 68 blows/ft at depth. Based on these values, together with the results of the dynamic penetration test, the density of the till is considered to range from compact at the surface to very dense at depth.

A piezometer was installed in the till stratum. The water level in the piezometer on July 12, 1968, 2 weeks after completion of boring, was at elevation 246, or about 5 feet above the river level. The poly tubing leading to the piezometer was blocked at elevation 230 and it is considered that this blockage is affecting the level in the piezometer. As the sandy silt till is in connection with the river at the bridge crossing, it is considered that the water level in the till at this site closely reflects the present river water level.

PROPOSED BRIDGE STRUCTURE

a) General

The existing bridge is a concrete structure of 42 foot span and about 15 foot width. The upstream wing walls of this structure have fallen in. The faces of the abutments have been badly eroded at and below normal high water and some erosion on the faces of the downstream wing walls have also taken place. It is understood that it is planned to replace this bridge at the same location with a wider structure of 55 foot span. The roadway grade at the bridge will be raised by about 2 feet.

b) Foundations

It is recommended that the abutments of the proposed bridge be founded on spread footings placed in the sandy silt till which underlies the site at about river bed level. To

provide protection against scour, the footings should be taken down at least 4 feet below the river bed.

The "N" values and the dynamic penetration resistances obtained within the till at and below proposed foundation level were generally in excess of 20 blows/ft. Based on these values, an allowable bearing pressure of 3 tons/sq.ft. may be used in design of footings founded in the sandy silt till. With this allowable bearing pressure, the settlement of the bridge abutments should be minor, provided precautions are taken during construction to prevent loosening of the granular soil at and below foundation grade as discussed below.

In the computation of sliding resistance between a rough concrete footing base and the undisturbed sandy silt subsoil, a coefficient of friction of 0.40, which is a limiting value, may be used in design.

Closed end abutments should be backfilled for a distance of at least 5 feet horizontally with a well compacted, free draining and non-frost-susceptible granular material. Provision should also be made for drainage from the backfill to prevent hydrostatic or ice pressure build up behind the walls. With full effective drainage of the backfill, a coefficient of lateral earth pressure at rest, K_0 , = 0.4 and a total unit weight of 135 lb/cu.ft. should be used for the compacted granular backfill in design of the abutments of a rigid frame structure. For a simply supported structure in which some movement of the top of the wall could be tolerated, an active earth pressure coefficient, K_a , = 0.3 may be used.

c) Construction Procedures

Providing construction is carried out when the creek level is near low water level, the excavation for the abutment footings will be about 5 or 6 feet below river water level. Some control of the groundwater will therefore be required for footing excavations in this essentially granular soil to prevent a reduction in the in situ density of the subsoil at and below foundation level. This control could be obtained by excavation within a steel sheet piled cofferdam, the sheeting being driven to a penetration below final excavation equal to the depth of the excavation below the water level.

Alternatively the ground water control for a drawdown

of some 5 or 6 feet could be accomplished by pumping from properly filtered sumps. The sumps could consist of perforated culvert pipe of about 12 inch diameter. The pipes should be installed vertically in an excavation adjacent to the foundation area and the pipes should be backfilled with coarse sand and gravel. Prior to excavation for the foundations, the water level should be lowered to below foundation grade by pumping from these sumps. In this case, dykes consisting of the relatively impervious glacial till material could be constructed on the river side of the excavation to divert the river flow.

To prevent loosening of the till surface once foundation grade is reached, it is recommended that the base of the footing excavation be immediately covered by a mud mat of crushed stone.

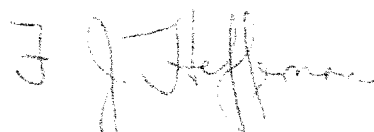
d) Approach Embankments

It is understood that the grade of the approach embankments will be raised a maximum of about 4 feet above present roadway level and some 15 feet above the floodplain or alluvium level. The approach embankment will also be widened considerably. Due to the granular and competent nature of the subsoil, there should be no overall stability problem with roadway approach embankments if raised to the height proposed, using $1\frac{1}{2}$ inch horizontal to 1 vertical side slopes, provided they are constructed of suitable fill material, properly compacted in place. The protection of the embankment slopes against erosion should be provided to some 2 feet above the maximum flood level. In this case, boulders graded from 1 foot to 6 inch diameter and dumped on the side slopes should provide adequate protection. Prior to widening the embankments, it is recommended that all surficial topsoil and organic matter should be stripped from beneath the proposed construction area.

We trust that this report contains sufficient information for your design purposes. If we can be of any further service to you on this project, please call us.

Yours very truly,

H. Q. GOLDER & ASSOCIATES LTD.



F. J. Heffernan, P. Eng.



FJH/ml
68766B
July, 1968.

GOLDER & ASSOCIATES

LIST OF ABBREVIATIONS

The abbreviations commonly employed on each "Record of Borehole," on the figures and in the text of the report, are as follows:

I. SAMPLE TYPES

AS auger sample
 CS chunk sample
 DO drive open
 DS Denison type sample
 FS foil sample
 RC rock core
 ST slotted tube
 TO thin-walled, open
 TP thin-walled, piston
 WS wash sample

II. PENETRATION RESISTANCES

Dynamic Penetration Resistance: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch diameter, 60 degree cone one foot, where the cone is attached to 'A' size drill rods and casing is not used.

Standard Penetration Resistance, *N*: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch drive open sampler one foot.

WH sampler advanced by static weight—weight, hammer
 PH sampler advanced by pressure—pressure, hydraulic
 PM sampler advanced by pressure—pressure, manual

III. SOIL DESCRIPTION

(a) Cohesionless Soils

| Relative Density | <i>N</i> , blows/ft. |
|------------------|----------------------|
| Very loose | 0 to 4 |
| Loose | 4 to 10 |
| Compact | 10 to 30 |
| Dense | 30 to 50 |
| Very dense | over 50 |

(b) Cohesive Soils

| Consistency | <i>c_u</i> , lb./sq. ft. |
|-------------|------------------------------------|
| Very soft | Less than 250 |
| Soft | 250 to 500 |
| Firm | 500 to 1,000 |
| Stiff | 1,000 to 2,000 |
| Very stiff | 2,000 to 4,000 |
| Hard | over 4,000 |

IV. SOIL TESTS

C consolidation test
 H hydrometer analysis
 M sieve analysis
 MH combined analysis, sieve and hydrometer¹
 Q undrained triaxial²
 R consolidated undrained triaxial²
 S drained triaxial
 U unconfined compression
 V field vane test

NOTES:

¹Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.

²Undrained triaxial tests in which pore pressures are measured are shown as \bar{Q} or \bar{R} .

LIST OF SYMBOLS

I. GENERAL

| | |
|---------------------------|---------------------------------------|
| π | $= 3.1416$ |
| e | $=$ base of natural logarithms 2.7183 |
| $\log_e a$ or $\ln a$ | natural logarithm of a |
| $\log_{10} a$ or $\log a$ | logarithm of a to base 10 |
| t | time |
| g | acceleration due to gravity |
| V | volume |
| W | weight |
| M | moment |
| F | factor of safety |

II. STRESS AND STRAIN

| | |
|-----------------|--|
| u | pore pressure |
| σ | normal stress |
| σ' | normal effective stress ($\bar{\sigma}$ is also used) |
| τ | shear stress |
| ϵ | linear strain |
| ϵ_{xy} | shear strain |
| ν | Poisson's ratio (μ is also used) |
| E | modulus of linear deformation (Young's modulus) |
| G | modulus of shear deformation |
| K | modulus of compressibility |
| η | coefficient of viscosity |

III. SOIL PROPERTIES

(a) Unit weight

| | |
|------------|---|
| γ | unit weight of soil (bulk density) |
| γ_s | unit weight of solid particles |
| γ_w | unit weight of water |
| γ_d | unit dry weight of soil (dry density) |
| γ' | unit weight of submerged soil |
| G_s | specific gravity of solid particles $G_s = \gamma_s / \gamma_w$ |
| e | void ratio |
| n | porosity |
| w | water content |
| S_r | degree of saturation |

(b) Consistency

| | |
|-----------|--|
| w_L | liquid limit |
| w_P | plastic limit |
| I_P | plasticity index |
| w_S | shrinkage limit |
| I_L | liquidity index $= (w - w_P) / I_P$ |
| I_C | consistency index $= (w_L - w) / I_P$ |
| e_{max} | void ratio in loosest state |
| e_{min} | void ratio in densest state |
| D_r | relative density $= (e_{max} - e) / (e_{max} - e_{min})$ |

(c) Permeability

| | |
|-----|-------------------------------|
| h | hydraulic head or potential |
| q | rate of discharge |
| v | velocity of flow |
| i | hydraulic gradient |
| k | coefficient of permeability |
| j | seepage force per unit volume |

(d) Consolidation (one-dimensional)

| | |
|-------|--|
| m_v | coefficient of volume change $= -\Delta e / (1+e) \Delta \sigma'$ |
| C_c | compression index $= -\Delta e / \Delta \log_{10} \sigma'$ |
| c_c | coefficient of consolidation |
| T_v | time factor $= c_v t / d^2$ (d , drainage path) |
| U | degree of consolidation |

(e) Shear strength

| | |
|----------|---|
| τ_f | shear strength |
| c' | effective cohesion intercept |
| ϕ' | effective angle of shearing resistance, or friction |
| c_u | apparent cohesion* |
| ϕ_u | apparent angle of shearing resistance, or friction |
| μ | coefficient of friction |
| S_i | sensitivity |

in terms of effective stress
 $\tau_f = c' + \sigma' \tan \phi'$

in terms of total stress
 $\tau_f = c_u + \sigma \tan \phi_u$

*For the case of a saturated cohesive soil, $\phi_u = 0$ and the undrained shear strength $\tau_f = c_u$ is taken as half the undrained compressive strength.

RECORD OF BOREHOLE 1

LOCATION See Figure BORING DATE JUNE 26, 1968 DATUM GEODETIC
 BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER 8X CASING
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT LB. DROP INCHES

| SOIL PROFILE | | SAMPLES | | | ELEVATION SCALE | DYNAMIC PENETRATION RESISTANCE BLOWS/FT. ----- | COEFFICIENT OF PERMEABILITY k, CM./SEC. | | | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION | |
|----------------|---|------------|--------|------|-----------------|---|--|------------------------|---|----------------------------|---|----------------|
| ELEV. DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | | BLOWS/FT. | SHEAR STRENGTH C _u , LB./SQ.FT. | WATER CONTENT, PERCENT | | | | |
| | | | | | | | | W _p | W | | | W _L |
| 251.3 | GROUND LEVEL | | | | | | | | | | | |
| 0.0 | | | | | | | | | | | | |
| | COMPACT GREY BROWN SAND AND GRAVEL, SOME COBBLES AND BOULDERS, TRACE OF SILT AND CLAY (EMBANKMENT FILL) | | | | | | | | | | | |
| 239.3 | | | | | | | | | | | | |
| 12.0 | STIFF GREY BROWN SILTY CLAY, SOME GRAVEL (ALLUVIUM) | | 2 | " | 22 | | | | | | | |
| 13.0 | | | 3 | " | 14 | | | | | | | |
| | COMPACT TO VERY DENSE BROWN CHANGING TO GREY AT ELEV. SANDY SILT, SOME GRAVEL, TRACE OF CLAY (SANDY SILT TILL) SILTY SAND LAYER AT ELEV | | 4 | " | 27 | | | | | | | |
| | | | 5 | " | 44 | | | | | | | |
| | | | 6 | " | 49 | | | | | | | |
| | | | 7 | " | 68 | | | | | | | |
| 220.3 | | | | | | | | | | | | |
| 31.0 | END OF HOLE | | | | | | | | | | | |

GROUND SURFACE
SURFACE SEAL

SAND FILL

POLY TUBING

M.H.

PIEZOMETER

W.L. IN
PIEZOMETER
AT ELEV. 246.3
JULY 12, 1968

POLY TUBING
BLOCKED AT
ELEV. 240.0

15 0 5
10

Percent axial strain at failure

GROUND SURFACE
SURFACE SEAL

SAND FILL

POLY TUBING

MH

PIEZOMETER

W.L. IN
PIEZOMETER
AT ELEV. 246.3
JULY 12, 1968

POLY TUBING
BLOCKED AT
ELEV. 240.0

15-0-5 Percent axial strain at failure

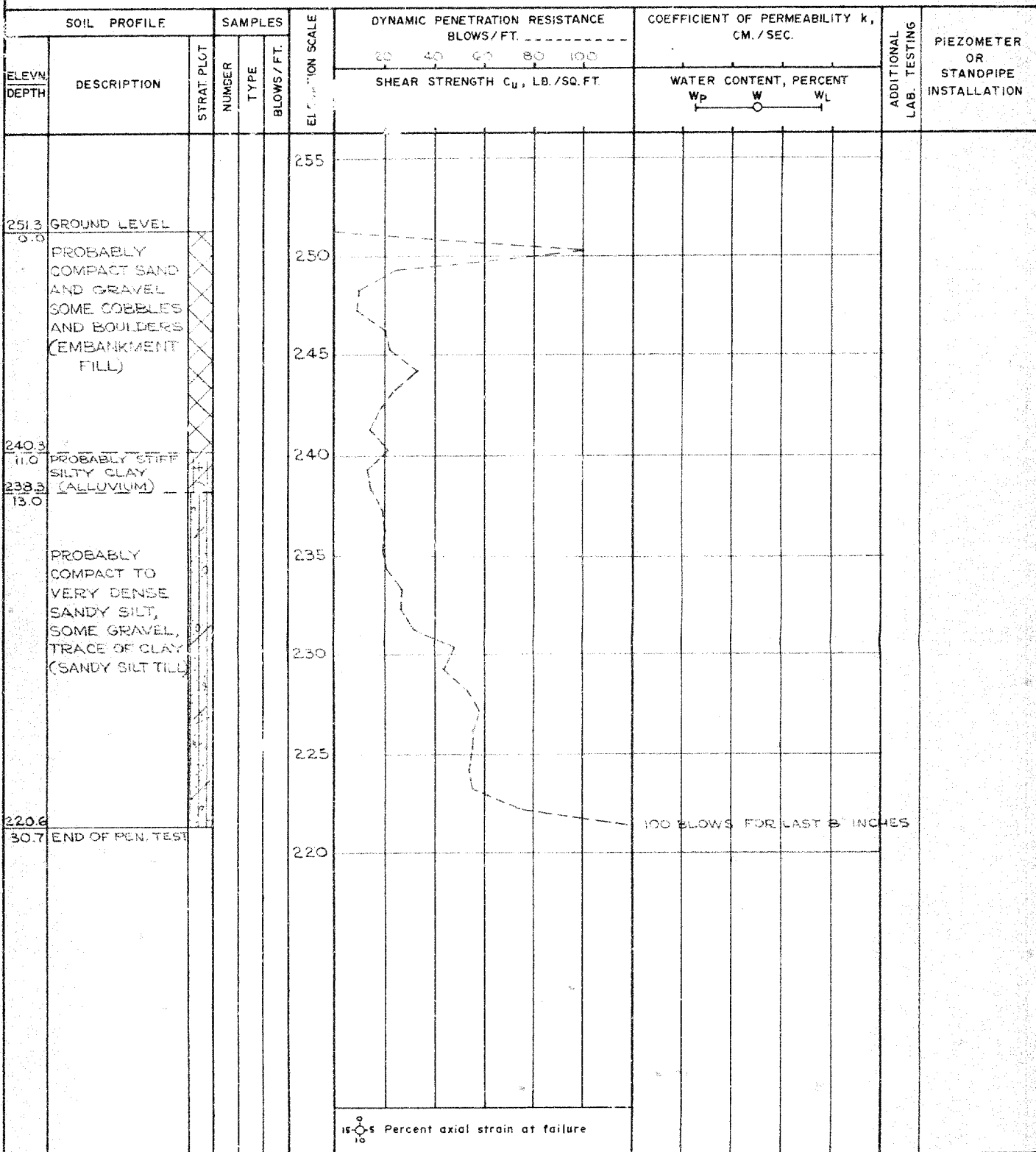
VERTICAL SCALE
1 INCH TO 5'

GOLDER & ASSOCIATES

DRAWN P.N.
CHECKED F.H.

PEN. TEST RECORD OF BOREHOLE A

LOCATION See Figure BORING DATE JUNE 27, 1968 DATUM GEODETIC
 BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER —
 SAMPLER HAMMER WEIGHT — LB. DROP — INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

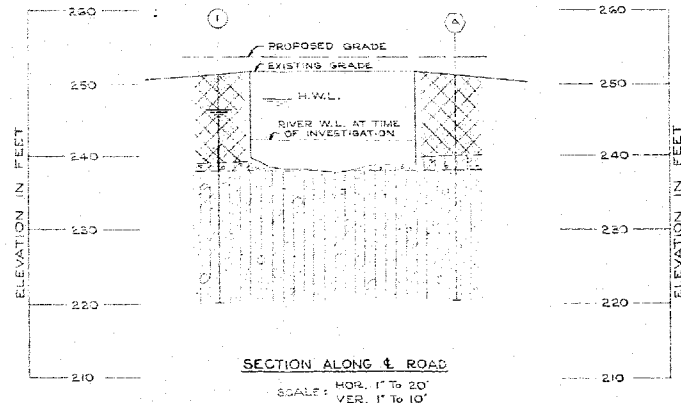
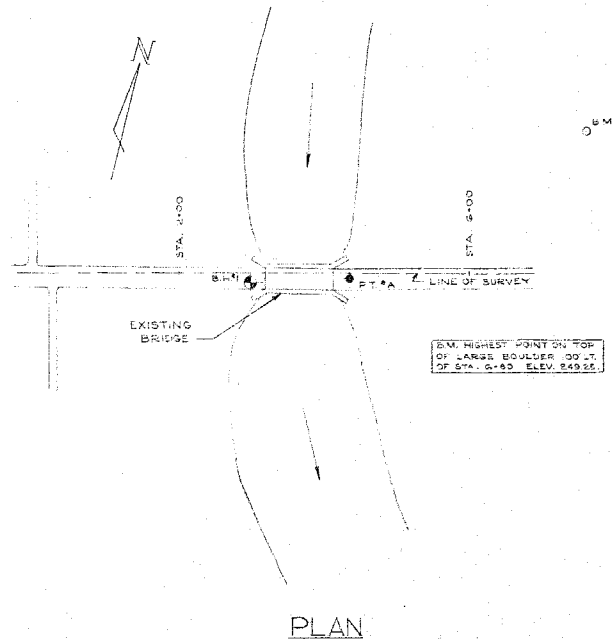


VERTICAL SCALE
 1 INCH TO 5'

GOLDER & ASSOCIATES

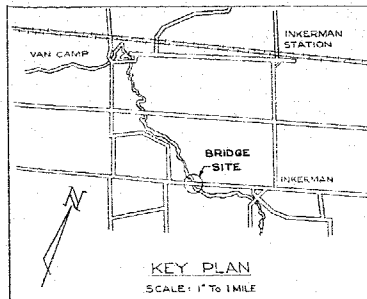
DRAWN D.N.
 CHECKED FIH

DEFECTS IN NEGATIVE DUE TO
 CONDITION OF ORIGINAL DOCUMENT



LEGEND

- BOREHOLE IN PLAN
- PENETRATION TEST IN PLAN
- BOREHOLE IN ELEVATION
- PENETRATION TEST IN ELEVATION
- WATER LEVEL IN ELEVATION



STRATIGRAPHY

- COMPACT GREY BROWN SAND AND GRAVEL, SOME COBBLES AND BOULDERS, TRACE OF SILT AND CLAY (EMBANKMENT FILL)
- STIFF GREY BROWN SILTY CLAY, SOME GRAVEL (ALLUVIUM)
- COMPACT TO VERY DENSE BROWN TO GREY SANDY SILT, SOME GRAVEL, TRACE OF CLAY (SANDY SILT TILL)

REFERENCE: DRAWING SUPPLIED BY
GRAHAM, BERMAN AND ASSOCIATES LTD.
JOB No. 1863, DATED JULY 10, 1968.

SPECIAL NOTE: DATA CONCERNING THE VARIOUS
SPREAD DATA WERE OBTAINED IN BOREHOLE LOC-
ATIONS ONLY. THE SOIL STRATIGRAPHY BETWEEN
BOREHOLES HAS BEEN INFERRED FROM GEOLOGICAL
EVIDENCE AND DO NOT VARY FROM THAT SHOWN.

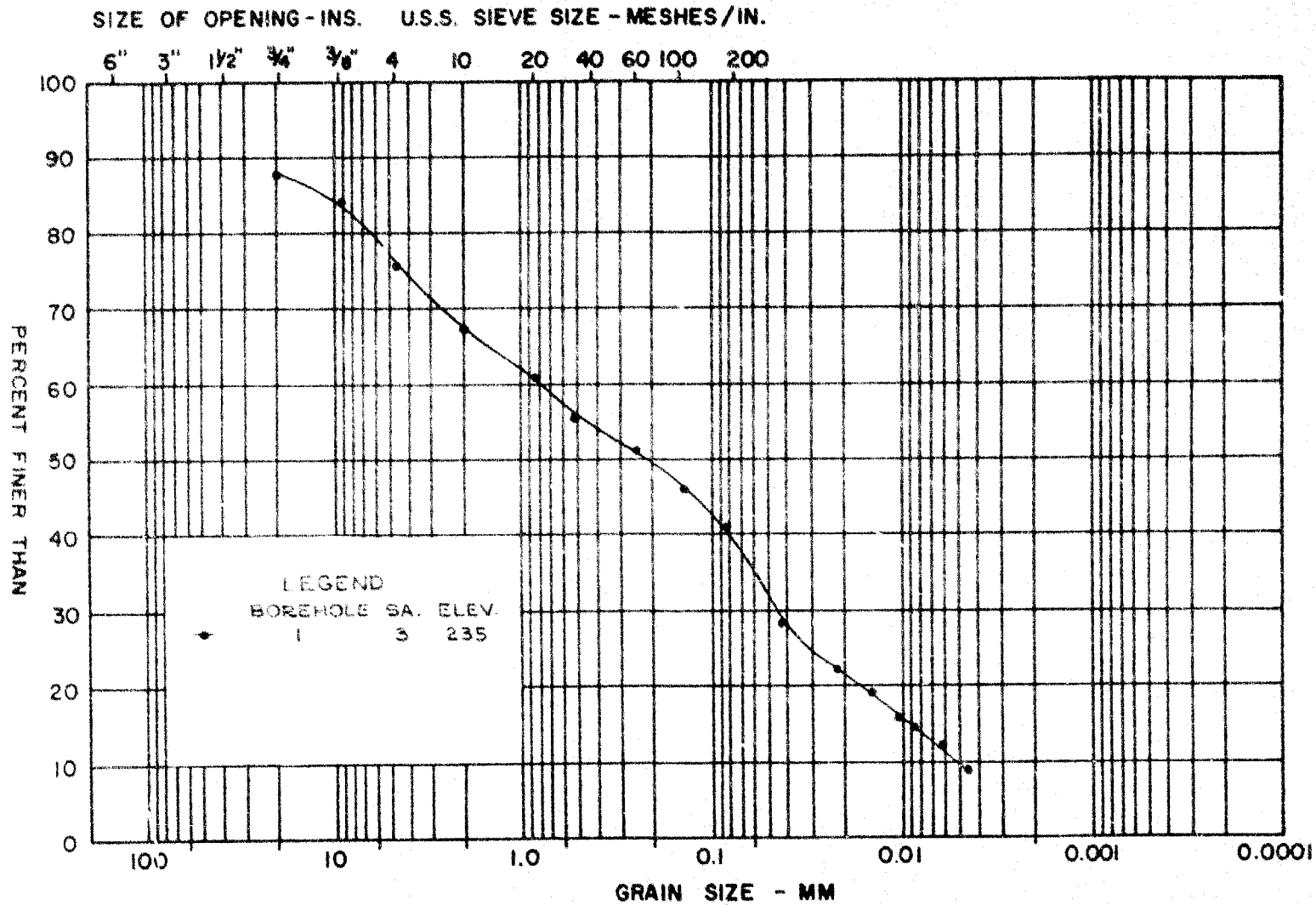
GOLDER & ASSOCIATES

Drawn: JULY 26, 1968.

Made ON
App 10 21
CART 25

EFFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

M.I.T. GRAIN SIZE SCALE



SANDY SILT TILL

GRAIN SIZE DISTRIBUTION

FIGURE 2

GOLDER & ASSOCIATES

| COBBLE SIZE | COARSE | MEDIUM | FINE | COARSE | MEDIUM | FINE | SILT SIZE | | CLAY SIZE |
|----------------|-------------|--------|------|-----------|--------|------|--------------|--|-----------|
| | GRAVEL SIZE | | | SAND SIZE | | | FINE GRAINED | | |